Structural Fire Performance of Concrete-Filled Steel Hollow Structural Sections: State-of-the-Art and Knowledge Gaps

STRUCTURAL FIRE PERFORMANCE OF CONCRETE-FILLED STEEL HOLLOW SECTIONS: STATE-OF-THE-ART AND KNOWLEDGE GAPS

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ABSTRACT

Concrete filled steel hollow structural (CFS) sections are an efficient, cost-effective, and architecturally attractive means by which to support large compressive loads in buildings, and they are thus increasingly popular among structural engineers and architects. Consisting of hollow steel tubes that are filled with one of a variety of types of concrete filling, CFS columns provide enhanced structural fire resistance and can, in many circumstances, be used with little or no applied fire protection insulation. The structural performance of modern CFS members, which may incorporate unreinforced or steel fibre reinforced high strength concrete infill, under the realistic thermal and loading scenarios that would be expected for a structural frame in a real building fire, is not fully known. This can lead to difficulties in design and construction which may discourage application of CFS members. This paper reviews current knowledge of the fire performance of CFS columns. A summary of fire scenarios, materials of construction, mechanical loading conditions, and fire protection systems considered in the available research (whether experimental or numerical) is provided, along with a discussion of gaps in knowledge and the possible consequences of these gaps for rational, performance-based design and analysis of structures incorporating CFS columns.

INTRODUCTION

Concrete filled steel hollow structural (CFS) sections are an architecturally, economically, and environmentally attractive means by which to support large compressive loads in buildings. These members consist of hollow steel tubes that are in filled with concrete to provide optimized load carrying capacity and structural fire resistance as compared with unfilled steel tubes. The concrete infill and the steel tube work together to yield several key benefits, both at ambient temperature and during a fire. The steel tube acts as stay-in-place formwork during casting of the concrete, thus reducing forming and stripping costs, it provides lateral confinement to the infill concrete which enhances the concrete's compressive strength and axial deformability, and it provides a smooth, rugged, architectural surface finish. The concrete infill drastically enhances the fire performance of the column by providing a heat sink and also by allowing the heated steel tube to shed its portion of the axial load to the concrete core when heated during a fire, thus providing adequate fire resistance without any applied fire protection to the tube in many cases. The infill concrete also enhances the steel tube's resistance to elastic local buckling[1].

Various types of concrete filling may be used in practice, including plain (unreinforced) concrete (PC), steel bar reinforced concrete (RC), and steel fibre reinforced concrete (FIB). A current trend in ambient temperature structural design of CFS columns is for the infill concrete to have a high-compressive strength of more than 60 MPa (up to 100 MPa in some cases). These members are now widely specified by architects and engineers and are increasingly being applied in the design and construction of multi-storey and high rise buildings as members within highly optimized structural frames where structural fire resistance ratings of two hours or more may be required[2].
Structural fire design guidance is available[1,3-8] for most common types of CFS columns, however, much of the available guidance was developed for conventional applications based on tests of

predominantly to short, concentrically-loaded, small-diameter columns in braced frames using normal strength. The structural performance of realistic, modern CFS columns, which may incorporate high strength and/or fibre reinforced concrete without any internal steel bar reinforcement or externally applied fire protection, under realistic thermal and loading scenarios – as would be expected for a structural frame in a real building fire – is not well known. This paper provides a review of available research on the fire performance of CFS columns covering experimental testing and computational modelling carried out worldwide. It focuses on the assumptions made in, and the uncertainties associated with, available research (whether experimental or numerical), with a view to highlighting research gaps and supporting safe application of CFS columns in realistic, performance-based design situations for multi-storey construction.

TEST DATA

Beginning in the mid 1950's and continuing to present day there have been more than 300 large scale standard fire tests carried out on CFS columns of various types. A reasonably comprehensive summary of the available test data and relevant test parameters is given in Tables 1 through 4. These tables divide the available data into tests on concentrically loaded unprotected columns (Table 1), eccentrically loaded unprotected columns (Table 2), concentrically loaded protected columns (Table 3), and eccentrically loaded protected columns (Table 4). In all cases the thermal exposure was based on a *standard* fire and all thermal exposures were thus similar or identical to the ISO 834 fire[9]. The main contributors to the available test database for concrete-filled SHS are the National Research Council of Canada (NRCC) and the Comité International pour le Dévelopment et l'Etude de la Construction Tubulaire (CIDECT). In some cases all the pertinent data are not available, most notably with respect to the specific details of the observed failure modes for many of the earlier tests. A key criticism of the available data is that they all imposed uniform heating in standard furnace tests, which have numerous well-documented shortcomings[10]. The key parameters noted in Tables 1 to 4 are discussed below.

Steel Tube Characteristics

The dimensions and strength of the steel tube section with respect to the concrete core play central roles in the fire performance of CFS columns. These parameters dictate the relative contributions of the steel tube and the infill concrete to the overall load carrying capacity of the column, both at ambient temperature and during fire. In general, unprotected columns, which rely more heavily on the steel tube, will tend to be more critical in fire since they lose a greater proportion of their strength due to heating of the external steel tube. However, various competing factors should also be considered, such as the fact that thinner walled tubes are more likely to buckle locally and this may affect both the effective length of a column during fire and its axial crushing strength. Interestingly, it appears that the specific factors leading to, and the consequences of, local buckling of the steel tube on the fire performance of CFS columns have received only limited direct research attention to date[11].

Standard tests performed to date have considered steel tube thicknesses ranging from 3.6-16 mm, with tube-to-infill cross-sectional area ratios as low as 0.9% and as high as 5.1%. Data from tests on circular sections from 121-600 mm in diameter are available in the literature, and on square sections ranging from 100-350 mm in minimum side length. However, the vast majority of tests (≈85%) have been on columns with a largest minimum dimension of less than 300 mm, and only a single test has ever been performed on a column over 478 mm in largest minimum dimension; in this case the load ratio (i.e. the ratio of the applied forces in fire conditions to the design load capacity of the member at room temperature) during testing was unrealistically low (about 0.2). Despite the obvious practical difficulties in performing realistic fire tests on members larger than 600 mm in diameter, the lack of fire test data for very large columns is currently a limiting factor in applying available design procedures[1]. This is somewhat puzzling, since it seems reasonable to assume that once the fundamental mechanics of CFS columns are understood and appropriately modelled, there is no obvious reason that larger sections should not be designed using available analysis tools. Steel strengths represented in the available fire test data range from 240 to 510 MPa.

The cross-sectional shape (circular or square) of the steel SHS plays two interrelated roles in the response of CFS columns during fire; one structural and one thermal. First, at ambient temperatures circular tubes are highly effective at uniformly confining the concrete core as axial loads are increased, so that the concrete is placed in a state of triaxial stress which increases both its strength and its axial-flexural deformability; square or rectangular columns provide less effective and non-uniform confinement with only minimal increases in strength but considerable enhancements in deformability. Loss of confinement due to excessive heating of the steel tube during a fire, which in addition to reductions in the strength and stiffness of the tube may also cause separation from the core due to differential thermal expansion, will result in a greater proportional loss of strength for a circular column than for a rectangular one (it should be noted however that the confining effects of the tube are often conservatively neglected in ambient design in any case). Second, circular sections exposed to uniform heating will heat up uniformly, whereas square sections will heat more rapidly at the corners, potentially inducing additional thermal stresses within the cross-section which could affect the response to heating. Both circular and square/rectangular concrete-filled SHS have been extensively studied in the available literature, with over a hundred standard furnace tests performed on each shape. However, the potential influence(s) of cross-sectional shape on the issues noted above, the observed failure modes, or the performance of applied fire protection have received little attention.

Concrete Infill Material

The type of concrete infill within the CFS column (PC, RC, or FIB) drastically affects its fire performance. Unprotected PC filled CFS columns fail at comparatively low loads when exposed to fire. Rapid loss of strength and stiffness of the fire exposed steel tube as temperatures increase cause loads to be shed to the concrete and, depending on the level of axial and flexural loads in the section, eventually lead to excessive local stresses in the concrete which cause failure as the concrete absorbs energy, the micro structure starts to deteriorate, creating micro cracks, and the concrete loses its continuity and thus its capacity to carry load[1]. PC filled CFS columns have particularly low fire resistance when load eccentricity, flexural loads, or second order effects play significant roles as described below. PC infill has been used in the majority of tests available in the literature (≈68%), whereas RC infill and FIB infill have been used in 25% and 7% of the tests, respectively.

Of the available fire tests on CFS columns, 79% have used concrete with f_c' < 50 MPa and only 8% have used f_c' > 70 MPa. This tendency toward lower concrete strengths reflects the fact that the bulk of the tests (≈63%) were performed prior to 1980, so that the tested concrete strengths were representative of mixes being used in construction at that time; these are not, however, reflective of current practice. Concrete specified in CFS columns in current multi-storey building designs tends toward 70 MPa or higher; this is clearly reflected in the literature by the recent emergence of studies focused specifically on the response to fire of CFS columns with f_c' up to 100MPa[12-15].

Not surprisingly, the introduction of internal steel reinforcement within the concrete core considerably increases the fire resistance of a CFS column, in particular when flexural effects are present. In addition to carrying a portion of the total loads on the column once the steel tube is heated, the internal steel reinforcement also acts to decrease the propagation and localization of cracks within the concrete and slows the loss of strength on further heating[16]. As is the case for conventional RC columns, the increase in fire performance depends on many factors, although the reinforcement ratio and amount of concrete cover to the internal reinforcement within the concrete core are key factors.

Core steel reinforcement ratios between 1.0-5.1% have been tested, although the vast majority (> 80%) have used between 1.0-3.0% with various internal layouts, typically using between four and eight longitudinal bars with square ties or steel spirals This is comparable to (perhaps slightly lower than) that which would typically be found in conventional concrete columns. Reinforcement ratios above 3.0% have been shown to provide comparatively little benefit for improved fire resistance[17].

While RC filled SHS columns perform well in fire and can typically be designed without any need for applied fire protection, there are many practical concerns associated with the placement of the internal steel cages which can be difficult, costly, and time consuming. Thus, RC infill is unfavoured in modern CFS column designs. There is a clear trend toward the use of PC infill which, although the least costly option and the most easily placed, considerably reduces fire resistance and can force the use of applied fire protection (bringing considerable additional costs and construction issues). One possible means to avoid having to use applied fire protection, first examined in the late 1970s[18] with limited success, but more recently studied in additional detail with increased success[19], is to use FIB infill. The advantage of FIB over PC infill is that suitably proportioned steel fibres within the concrete arrest the propagation of micro cracks and improve the continuity of the concrete core and its ability to carry load. Furthermore, the fibres enhance the tensile strength of the infill concrete, potentially allowing it to carry tensile forces due to small flexural effects (although this has yet to be experimental confirmed). The fibres also slightly increase the compressive strength of the infill[1]. Research studies at NRCC have shown that FIB infill can provide fire resistance values which are comparable to those of RC filled SHS columns[20], although this has only been shown for concentric loading for a single steel fibre type and volume content (1.8% by mass). FIB infill reduces the likelihood of internal concrete spalling and separation and thus alleviates the potential problems of lack of continuity in the concrete. Hybrid fibre reinforced concrete (incorporating both polypropylene and steel fibres) may further enhance performance in fire. The mechanics of FIB infill CFS columns in fire remain poorly understood and additional research is needed.

The type of aggregate used in the concrete also plays a role during fire. Different aggregates may result in an order of magnitude difference in the coefficient of thermal expansion of the core. This may impact heat transfer within the section and the formation and size of the air gap. It may also affect the transfer of load from the steel tube to the concrete core, and hence the column’s deformation and ultimate failure mode. Only limited research has considered the possible effect(s) of aggregate type on CFS columns. For instance, an NRCC study[21] showed that a siliceous aggregate RC infilled CFS column tested at a load ratio of 0.58 had half the fire endurance of an equivalent CFS column with carbonate aggregate infill tested at the same load ratio. Considerations around aggregate types may become important in the future as sustainability concerns force contractors to use locally sourced materials.

Slenderness & Rotational Restraint

The relative slenderness and end fixity of the tested specimens is crucial when considering their response to fire, particularly in terms of their observed failure modes since slender columns are more likely to fail by *global* buckling whereas short columns will fail by *local* buckling and/or crushing of the core. Column lengths between 760-5800 mm are represented in the literature, although the vast majority (≈83%) are between 3030 and 3810 mm. This is due to the size of available standard testing facilities globally. The lack of data from realistic fire tests of slender CFS columns is currently claimed to limit their application in many applications[1]. However, the most slender CFS columns reported in the literature have non-dimensional slenderness (calculated according to Eurocode procedures[3-5]) of about 1.8. This slenderness is well within the practical range for CFS columns that are likely to be considered in all but a small minority of practical design situations.

Several column end fixity combinations are represented in the literature and are given as: fixed-fixed (FF), pinned-fixed (PF), and pinned-pinned (PP). The majority of tests (≈64%) have been on FF members, although it is worth noting that the true fixity during testing is never perfect and is probably not known. It should also be noted that columns are rarely heated over their entire height during furnace testing (for instance, in the NRCC testing furnace columns are heated over only 80% of their total length[19-21] with the ends insulated). This has potentially important implications, particularly for FF and PF columns when relating the non-dimensional slenderness at ambient conditions to the effective slenderness of the column during a fire test. Unheated regions will maintain their full flexural stiffness during a fire test, which artificially reduces the assumed effective slenderness during the test as compared with a column in a real building. Furthermore, there is compelling evidence, from non-standard furnace tests performed on CFS columns which included load introduction regions with beams framing into the columns during the tests, that end fixity, load introduction, axial load ratio, and steel tube thickness all influence both the likelihood and location of local buckling of the steel tube during fire – the location of the local buckle being the primary factor dictating the effective length of the column during a fire[22]. Thus, FF and FP furnace tests may be unconservative with respect to the true effective length of CFS columns during fire unless the true end fixities of the tested columns are accounted for in considerable detail. This issue is particularly noteworthy given that Eurocode[4] provisions permit the effective lengths of columns in non-sway frames to be taken as 0.5 during fire if the columns are continuous across multiple floors, whereas certain test data suggest that local buckling of the steel tube may lead (unconservatively) to an effective length of 1.0 times the storey height regardless of the end fixity condition or continuity[22].

Load Eccentricity & Bending

The relative importance of load eccentricity and bending depends predominantly on the type of concrete infill. The majority of available tests (>80%) have been on CFS columns under concentric load. Intentionally applied load eccentricity ratios between 2.5% and 150% are present in the literature. However, the only rational way to test the specific impacts of load eccentricity for various types of concrete infill would be to test identical CFS columns with different initial load eccentricities but at the same load ratio; such comparative data are scarce.

The available data clearly show that CFS columns filled with PC are highly sensitive to load eccentricity, and that they suffer large reductions in fire resistance times under loads of increasing eccentricity (all other factors being equal). This is clearly due to the fact that PC infill is severely limited in its ability to carry flexural loads once the steel tube heats and sheds its load to the concrete core. Unprotected CFS columns with PC infill are generally not used where load eccentricity or bending (including slenderness effects) are expected during a fire. When identical CFS columns with PC infill are tested under different initial load eccentricities but at the same fire test load ratio, the specific impact of eccentricity is less severe, although the available data are in many cases highly contradictory. For unprotected PC filled CFS columns with identical load ratios the available data[12,13] suggest that initial eccentricity ratios as high as 30% may have no obvious detrimental effect on fire resistance (albeit with fire resistances of less than 30 min in all cases). For fire protected PC filled CFS columns however, limited data[23] show that eccentricity ratios of only 10% may cause reductions in fire resistance of up to 40% (with a fire resistance of 166-188mins for concentric loading). For unprotected CFS columns with RC infill, no negative influence of eccentricity or bending is expected within the practical range of internal steel reinforcement ratios for columns with the same load ratio[24]. While research has suggested that use of FIB infill can improve the fire performance of concentrically loaded CFS columns with unreinforced concrete infill, the benefits of FIB infill for columns with eccentric loads and/or bending have not been properly investigated.

Load Ratio

In practice, the load ratio for a structural member typically lies somewhere in the range of 0.3 to 0.5[25], and in some cases up to 0.6 depending on a multitude of ambient design considerations. The fire resistance of any type of column is explicitly linked to the sustained load applied during testing, with higher load ratios leading to lower fire resistance ratings. Load ratios of less than 0.3 (≈30% of available tests) are likely to be unrealistically low, and greater than 0.6 (≈21% of available tests) are likely to be unrealistically high. Furthermore, it is important to consider the eccentricity of the applied load and the resulting reduction in nominal strength at ambient (due to axial-flexural interaction) when quoting the load ratio imposed during a fire test.

Table 1. UNPROTECTED CONCENTRICRICALLY LOADED CFS COLUMNS

Researchers: Name[Ref]	Date	Specimen details: Length	End Conditions		Load Ratio		Steel Tube: Section Shape		Section Size	Wall Thickness	Steel Strength	Concrete: Concrete Type		Strength: 28 - Day	Test	A_s/A_c Ratio	Cover	Failure: Time C	Time S	Mode LB	GB	C
		m							*mm*	*mm*	*MPa*			*MPa*	*MPa*		*mm*	*min*	*min*			
NRCC[15,16,19,21,29,34]	1982-95	3.81	PP	5	<0.3	40	C	51	141 - 406	4.8 - 12.7	300 - 350	PC	45	24 - 91	12 - 107	-	-	48-294	62-131		24	21
			PF	-	<0.6	28	S	22	150 - 305	5 - 12.7	300 - 419	RC	12	38 - 82	38 - 93	2.1 - 2.5%	40	43-188	39-212		6	6
			FF	68	>0.6	4						FIB	16	42 - 90	39 - 100	1.77%	-	65-259	60-128		7	9
CIDECT[#][17,18,41]	1954-76	3.6	PP	-	<0.3	7	C	18	121 - 600	3.6-16	240-420	PC	36	7-95	30-52	-	-	12-198	16-458	??		
		to	PF	-	<0.6	25	S	35	140 - 330	3.6-8	300-429	RC	14	24-37	32-51	1.3-3.7%	25-50	-	25-192			
		4.8	FF	33	>0.6	7						FIB	3	47-52	50-56	4.30%	-	-	19-80			
CIDECT[14,18]	1977-2000	0.8	PP	6	<0.3	34	C	19	159-406	3.6-12.5	286-410	PC	38	31-96	34-52	-	-	28-102	30-104	??		
		to	PF	4	<0.6	20	S	35	150 - 350	3.6-10	243-510	RC	12	32-96	36-50	1 - 2.6%	35-43	71-134	51-135			
		5.8	FF	44	>0.6	-						FIB	4	40-98	48-58	4.30%	-	-	55-81			
Han[13,42]	2003	3.81	PP	6	>0.6	6	C	4	150-478	4.6-8	259-381	PC	6	40-69	49	-	-	20-29	16-21	1	4	1
							S	2	150-200¢	8	341											
Sakumoto[23]	1993	3.5	PP	1	<0.3	1	S	1	300	9	358	PC	1	??	37.5	-	-	-	33	1*		1*
Kim[45]	2005	3.5	PP	20	<0.6	20	C	10	319-406	7-9	304-311	PC	20	28 - 38	??	-	-	28-150	44-160	??		
							S	10	300-350	9	363											
Lu[12]	1993	0.76	FF	4	<0.3	2	S	4	150-200	5-6	467-486	PC	4	90-99	??	-	-	-	26-92	4*		4*
					<0.6	2																

Table 2. UNPROTECTED ECCENTRICRICALLY LOADED CFS COLUMNS

Researchers: Name[Ref]	Date	Specimen details: Length	End Conditions		Load Ratio		Steel Tube: Section Shape		Section Size	Wall Thickness	Steel Strength	Concrete: Concrete Type		Strength: 28 - Day	Test	A_s/A_c Ratio	Cover	Failure: Time C	Time S	Mode LB	GB	C
		m							*mm*	*mm*	*MPa*			*MPa*	*MPa*		*mm*	*min*	*min*			
NRCC[16,34]	1990-94	3.81	PP	3	<0.3	2	C	1	219	8.2	350	PC	1	24.3	31.9	-	-	33	-	-	1	-
					<0.6	1	S	2	300	8	394	RC	2	40.7	43.8	5.07%	40	-	58-126	-	2	-
		Eccentricity ratio			15-40%																	
CIDECT[18,24,43,44]	1977-82 & 2001	3.03	PP	36	<0.3	2	C	5	133-356	4-6	235-383	PC	6	30-43	32-44	-	-	33-69	24-55	??		
		to			<0.6	20	S	31	110-300	4-12.5	394	RC	30	31-75	36-38	0.9-4.4%	15-43	45-56	22-92			
		5.2			>0.6	6																
		Eccentricity ratio			2.5-150%																	
Han[13,42]	2003	3.81	PP	6	>0.6	6	C	4	219-478	4.6-8	293-381	PC	6	40-69	49	-	-	7-32	20-24	-	4	2
							S	2	150-200¢	8	341											
		Eccentricity ratio			15-30%																	
Lu[12]	2003	0.76	FF	2	<0.6	2	S	2	150-200	467-486	394	PC	2	90-99	??	-	-	-	43-55	2*	-	2*
		Eccentricity ratio			12.5-17%																	

Tables 1 & 2, notations : ¢ - rectangular (300 x ¢); * *local buckling occurred first followed by crushing of the concrete;* # Specimen details not fully known

Table 3. PROTECTED CONCENTRICRICALLY LOADED CFS COLUMNS

Researchers		**Specimen details**					**Steel Tube**					**Concrete**						**Failure**				
Name[Ref]	Date	Length	End Conditions		Load Ratio		Section Shape		Section Size	Wall Thickness	Steel Strength	Concrete Type		Strength		A_s/A_c Ratio	Cover	Time		Mode		
														28 - Day	Test			C	S	LB	GB	C
		m							*mm*	*mm*	*MPa*			*MPa*	*MPa*		*mm*	*min*	*min*			
CIDECT[18]	1971-75	3.6	PP	2	<0.3	2	C	2	168-219	3.6-12.5	300	PC	32	18-51	??	-	-	60-90	35-290	??		
			FF	33	<0.6	14	S	33	140-225	3.6-8	355-360	RC	3	45-47	41 - 55	1.1-3.7%	20	-	35-130			
					>0.6	25																
		Intumescent Paint - 6 (1.4 - 2 kg/m^2), 6 - Rock Wool, 2 - Liquid Stone¥, 9 - Vermiculite Boards, 3 - Plaster, 3 - Plaster Shells, 2 - Alphapan§, 4 - Asbestos Cement																				
Sakumoto[23]	1993	3.5	PP	4	<0.3	4	S	4	300	9	358-361	PC	4	??	38	-	-	-	166-194	4*		4*
		Ceramic Board – 3 Intumescent Paint -1 (1.25 kg/m2)																				
Han[13,42]	2003	3.81	PP	11	>0.6	11	C	5	150-478	4.6-8	259-381	PC	11	18-69	19-49	-	-	120-196	78-169		10	1
							S	6	150-350¢	5.3-8	246-341											
		Intumescent Paint - 11 (4.4 - 10 kg/m^2)																				
Edwards[26]	1997	3.6	FF	6	<0.6	5	C	2	168-324	6.3	306-321	PC	6	34-43	43-48	-	-	115-166	102-146	??		
					>0.6	1	S	4	150-300	6.3-16	331-375											
		Intumescent Paint - 6 (0.8 - 1.1 kg/m^2)																				

¢ - 4 rectangular, 2 square   * *local buckling occurred first followed by crushing of the concrete*

¥ - Liquid Stone protection consisted of vermiculite particles mixed with a synthetic stone produced by a reaction of calcite and portlandite[16]

§ - Alphapan is a form of protection consisting of panels cut from plates made of agglomerated rock fibres[16]

Table 4. PROTECTED ECCENTRICRICALLY LOADED CFS COLUMNS

Researchers		**Specimen details**					**Steel Tube**					**Concrete**						**Failure**				
Name[Ref]	Date	Length	End Conditions		Load Ratio		Section Shape		Section Size	Wall Thickness	Steel Strength	Concrete Type		Strength		A_s/A_c Ratio	Cover	Time		Mode		
														28 - Day	Test			C	S	LB	GB	C
		m							*mm*	*mm*	*MPa*			*MPa*	*MPa*		*mm*	*min*	*min*			
Sakumoto[23]	1993	3.5	PP	4	<0.3	3	S	4	300	9	358-361	PC	4	??	38	-	-	-	88-148	-	4	-
					<0.6	1																
		Ceramic Board – 3			Intumescent Paint -1 (1.25 kg/m^2)							Eccentricity ratio 10%										
Han[13,42]	2003	3.81	PP	1	>0.6	1	S	1	350	8	284	PC	1	18	19	-	-	-	108	-	-	1
		Intumescent Paint -1 (2.8 kg/m^2)										Eccentricity ratio 30%										

Abbreviations used in tables: PP – Pinned-pined, PF – Pinned-fixed, FF – Fixed-fixed , C – Circular, S – Square, PC – Plain concrete, RC – Reinforced concrete, FIB – Fibre reinforced concrete, As – Cross-sectional area of steel, Ac – Cross-sectional area of concrete, LB – Local buckling, GB – Global buckling, C – Crushing.

Failure Modes

The four typical stages of deformation of a concentrically loaded CFS column in fire are well documented in the literature[6]. In Stage I, the steel tube heats up it expands both in the horizontal (radial) and vertical (longitudinal) directions. The steel, having a higher coefficient of thermal expansion and heating more rapidly, expands at a faster rate than the concrete infill and this can create a gap between the steel tube and the infill and allow the steel to expand unrestrained. The precise consequences of this expansion in the hoop direction are not well known, although it appears that there are both thermal effects (i.e. reduction of heat transfer to the concrete and effective insulation of the back face of the steel tube) and structural effects (i.e. removal of support against local buckling of the tube side-wall). In the axial direction thermal expansion of the tube causes it to take more of the load as it expands but is restrained by the floors above. This longitudinal expansion continues until the tube takes so much of the load that it yields locally in compression and rapidly shortens and transfers load back to the core (Stage II). Provided that the column remains stable during this contraction (which is *not* assured) the load will continue to be carried by the cooler concrete core with only minor changes in column length as the fire continues (Stage III). The core continues to carry the load until there is sufficient degradation of the concrete that the load can no longer be supported and the column fails (Stage IV).

It is unfortunate that many of the available testing reports from fire tests on CFS columns devote relatively little attention to describing the observed failure modes in any significant detail, since this information is of fundamental importance in understanding the mechanics at play during fire. Tests have generally been grouped into two broad categories: buckling and crushing. *Global* buckling failures occur when three locations of little to no rotational restraint (hinges) develop a collapse mechanism in a column and large lateral deflection of the column occurs. The hinge locations are invariably associated with areas of *local* buckling of the steel tube, and it should be noted that the factors influencing the formation of these hinge regions (including load introduction, rotational restraint, inter-storey effects, localized heating) remain poorly understood[22]. Buckling failures are more prevalent in slender columns with eccentrically applied loads. Crushing failures occur where the degradation of the core concrete's compressive strength and integrity is sufficient that the load can no longer be supported. Such failures are typically accompanied by *local* 'elephant's foot' buckling of the steel tube at the crushing location. This has apparently been observed coincident with the majority of crushing failures; however the location of these local buckles is not consistent from test to test. Clearly, the buckle location will impact the crushing and global buckling capacities and should be clearly reported in future tests. Of the available tests for which failure modes are clearly quoted (≈96 tests), 47% are stated as buckling and 53% as crushing.

Finally, it must be noted that not all CFS columns are able to transition from Stage II to Stage III, and some CFS columns fail shortly after first yielding or local buckling of the SHS tube. As discussed by Wang and Orton[6], whether a CFS column is able to pass through all four stages of deformation depends on many factors, although risk factors include slenderness, low internal reinforcement ratio, high applied load, load eccentricity, and (for reasons unknown) stiff rotational restraint 'at the top'. The fact that Wang and Orton[6] state that there is currently no simple method to identify CFS columns that are *not* able to go through all four stages is clear evidence of a fundamentally incomplete understanding of the mechanics and interactions leading to failure.

Applied Fire Protection

As noted above, PC infilled CFS columns may require some kind of applied fire protection to achieve required fire resistance ratings in some applications. While spray-applied and board systems of fire protection have been applied to CFS columns, intumescent coatings are by far the preferred method of fire protection for these types of members. Many different, proprietary intumescent paints are available, and it is virtually impossible to make generalizations regarding their thermal insulation characteristics. While many hundreds (perhaps thousands) of certification tests have been performed on hollow SHS tubes protected with intumescent paints, relatively few such tests are available on CFS columns (particularly under load).

Approximately 24 tests are available in the literature that report on the use of intumescent protection in structural fire tests of CFS columns. These tests show that the coatings dramatically improve fire performance and are particularly of interest for achieving fire resistances of more than 30 minutes, although the evolution of thermal protection and its influence on the temperature profile across the section remain poorly understood. For instance, temperature gradient along the developing char, which will be directly related to the heat input as a result of the substrate's thermal response, can affect the evolution of an optimum insulating char layer, and most currently available intumescent paints have been carefully optimized to perform on unfilled steel sections or profiles rather than CFS columns. For any intumescent coating to function properly in a fire, the substrate needs to heat up at the correct rate, which demands conformance tests specific to CFS columns which exist only for specific products[26]. The heat sink effect of infill concrete can possibly be translated to a lower effective section factor. The main criterion for any intumescent formulation in this application is to be able to cover section factors of about 300/m as well as a lower range of section factors that could include CFS columns. However, it should be noted that at very low section factors certain intumescent products may crack and debond prior to intumescing. There is a lack of reliable thermal property data for intumescent systems[6], and very little is known about the influence of the heat sink effects of the concrete infill, or the formation of a gap between the steel and the concrete, on the evolution or stickability of the intumescent char.

COMPUTATIONAL MODELLING

Many modelling approaches have been used in an attempt to predict the fire resistance of CFS columns. The motivation for these models is two fold. First and foremost, a suitably validated model can be used to perform parametric studies on various column parameters and develop simple analytical formulae and procedures for column design, without the need to test large numbers of specimens. Second, suitably validated 3D finite element (FE) models can be used to study (with limited verifiability in many cases) the specific impacts of key issues (e.g. non-standard heating regimes, air gap formation, local buckling, longitudinal slip between the concrete and the steel tube, etc) which cannot be easily captured using simple sectional analysis models. It is noteworthy that all of the computational modelling approaches discussed below depend on user inputs for a wide variety of parameters for which limited guidance is available (for instance, the resultant emissivity in fire is taken anywhere between 0.5-1.0 in the quoted studies, typically with little or no justification), and have been validated by comparison against 'selected' test data. Full statistical comparisons of the respective modelling approaches against the full database of available test results are not available.

Simple Crushing Analyses

The simplest models presented in the literature predict only the crushing strength of CFS columns[27,28]. These models simply apply stress-strain curves for the columns' constituent materials at elevated temperature (taken from any number of available sources) and assume that the thermal and structural behaviour of the member is uncoupled, that there is perfect bond between the steel tube and the infill, that no gaps form between the tube and the concrete, that no slip occurs between the tube and the concrete, and that neither local nor global buckling need be considered; all of these assumptions are known to be false, but the degree to which they influence the models' predictive ability is not clearly known. Such an analysis is only ever potentially appropriate for stocky columns, and based on the most-observed failure modes in experiments it would seem that such approaches are indefensible in most cases. Indeed, in a study by Chung et al.[27] using this approach the model over-predicted both the temperatures and the failure times in every comparison against tests.

Cross-Sectional Equilibrium Approaches

Using an approach originating at NRCC in the 1970s, several models have taken a relatively simple approach based on a cross-sectional equilibrium analysis[13,29]. The column's cross-section is divided into annular or square elements and sectional equilibrium at mid height is used through an iterative analysis to develop curves of capacity versus time of fire exposure. These models assume that the concrete has no tensile strength, plane sections remain plane, there is perfect bond between steel and concrete (and thus no slip, no air gap, and no local buckling), that there is no composite action between the steel and the concrete, and no concrete confinement due to the steel tube. It is further assumed that effective length of the fixed-fixed column remains uniform throughout the heating at $0.7L_{cr}$ (essentially arbitrarily but chosen to match test data from the NRC furnace), and that the deflected shape of the column is sinusoidal – therefore prescribing the failure mechanism for the column as one with a hinge at mid-height. Comparison against results from selected NRCC tests has shown this approach to conservatively predict fire resistance. These models have fallen out of favour due to the accessibility of finite element software.

Custom Finite Element Packages

Several custom FE packages, developed specifically for structural fire analysis and incorporating varying degrees of complexity, have been applied to CFS columns. This includes work by Schaumann et al.[30] (using BoFIRE), Kodur and Fike[31] (using SAFIR), and Renaud[32] (using the purpose-built software SISMEF). These analyses all differ in many subtle respects which are not important for the current discussion. What *is* important at present is that neither the BoFIRE nor SAFIR analyses apparently included the effects (thermal or structural) of gap formation, slip between the SHS tube and the concrete, concrete confinement, or local buckling, despite the fact that the authors highlighted the potential importance of these issues.

Renaud's[32] comprehensive analysis *does* consider the thermal impacts of gap formation (albeit by imposing a predefined thermal resistance so as to match test observations) as well as the structural impacts of slip between the steel tube and the infill concrete (using a special 'composite bar' connection element). The analysis appears not to consider local buckling of the SHS tube. Renaud's analysis is validated against only 33 tests (of the 300+ tests available) and it is not clear how these specific tests were selected. Nonetheless, the SISMEF analysis 'appropriately' predicted the response of the CFS columns considered. A notable conclusion of Renaud's study is that slip appears to play an important role, particularly within the first 30 minutes of a standard fire test for CFS columns with PC infill or for CFS columns with applied load eccentricity, bending, or slenderness effects.

General Purpose Finite Element Models

Several studies have used widely available general purpose FE packages to perform structural fire analyses of CFS columns. Again, the issue of primary interest for the current discussion is the factors that have been neglected in the analyses. For instance, Zha[33] presents a 3D FE model of a circular CFS column exposed to a standard fire using DYNA3D, although again the model apparently neglects gap formation, slip, concrete confinement, and local buckling, and is validated only against the tabular design approach given in Eurocode 4[4] rather than against experimental data. Hong and Varma[34] used ABAQUS to model the standard fire behaviour of CFS columns and, while ignoring the influence of gap formation on the thermal response of the sections, included the effects of slip and local buckling in their analysis by artificially de-bonding the steel tube over a prescribed length near the column mid height. The model was validated against 15 tests available in the literature, but again it is unclear why these specific tests were chosen. This study confirmed that the effects of local buckling and slip are more important for columns which experience bending. Espinós et al.[35] also used ABAQUS to model CFS columns, neglecting gap formation, confinement, slip, and apparently local buckling, and verified their model against only eight experimental results, none of which had load ratios over 0.3.

The most advanced 3D FE modelling presented to date is presented by Ding and Wang[39] using ANSYS. This study carefully considered the potential thermal and structural impacts of gap formation as well as slip between the steel tube and the infill concrete and local buckling of the steel tube. The thermal influence of an air gap was modelled by arbitrarily assuming a constant air gap of 1 mm with an assumed associated thermal resistance imposed to match selected tests available in the literature.

The resulting thermal analysis indicated that the accuracy of temperature prediction in CFS columns in fire can be noticeably improved by accounting for the formation of an air gap. Given the number of parameters upon which the formation of an air gap depends, research is needed to understand and model this process for the range steel sections and concrete infill materials currently used in practice. Slip was considered in Ding and Wang's analysis using 3D surface-to-surface contact elements and a Coulomb friction model. Interestingly, the results of parametric studies to investigate the potential effects of slip on the lateral deflection response and time to failure of the columns indicated that the effects were minor. On the basis of their work, Ding and Wang[39] concluded that, for the columns examined in their study, it was not absolutely essential to include slip in the analysis, although slightly better results were obtained when slip was included, that the specific properties of the bond-slip response were of little significance as long as slip was included, and that introducing an air gap improved the accuracy of the thermal analysis and hence the structural performance predictions (although not introducing an air gap is likely to be conservative for unprotected CFS columns).

Other models reported in the literature include neural network[36] and Rankine[37] approaches. However, these methods are not expected to capture important subtleties of CFS columns in fire.

KNOWLEDGE GAPS

1. Fire Scenario

Current fire design procedures for CFS columns are based on standard furnace testing which has uniquely used standard fires. This is clearly inaccurate to model real fires[10], and advanced structural fire engineering solutions thus typically impose parametric design fires in a performance-based environment; data on the performance of CFS columns in design fire scenarios, which notably include a cooling phase, are not currently available. Real fires (localized or travelling) may also impose non-uniform heating which may induce column curvatures and the formation of plastic hinges or thermal curvatures leading to secondary moments in real structures. This may also be important for CFS columns forming part of a compartment wall or building façade, where one-sided heating may occur.

2. Materials of Construction

Most testing and modelling to date has focused on normal strength concrete infill whereas current practice is to use higher strengths. High strength concrete is known to be prone to spalling, to suffer proportionally greater losses in compressive strength on heating, and to display lower dilatency on loading, all of which may affect its response to loading in fire. Few tests have been performed on such columns (either constitutive, thermo-mechanical, or full-scale). Research[19,20] has suggested that FIB infill (whether normal strength or high strength) can provide similar fire resistance as RC infill (under concentric loading) although only limited data are available and the mechanisms of the improved response are neither confirmed nor understood, particularly when flexural effects are present.

3. Sectional Properties & Response

The effects of differential thermal expansion and gap formation on the heat transfer within, and structural response of, CFS columns needs to be better understood, both for protected and unprotected columns. The size and timing of gap formation has been shown to affect heat transfer calculations for CFS columns and may affect the evolution and effectiveness of applied intumescent fire protection. The cross-sectional size of a CFS column may influence the mechanical and thermal response of the column, particularly when intumescents are used (smaller sections tend to experience 'stickability' reductions). Column sizes being used commonly in high rise buildings are typically in excess of 600mm and can exceed 1600mm, with plate thickness of 25mm and more. No testing has been done (or is foreseeable) on columns of this size, so that a fundamental understanding of the underlying mechanics is needed to extend models and develop defensible designs. Finally, the bond-slip between infill concrete and steel tube has received relatively little attention but clearly has relevance for load introduction of when bending is present.

4. Mechanical Loading during Fire (Full Frame Response)

While a few tests with eccentric loading have been reported, very little information is available for the most practically interesting cases of unprotected FIB infill and protected FIB and PC infill columns. For perimeter/edge elements in steel frames and Diagrid frames, the potential effects of bending moments on CFS columns and the formation and location of plastic hinges in the fire limit state need to be rationally assessed, particularly for unbraced structures. The appropriate effective length of CFS columns in fire has received considerable research attention[38], yet available guidance (which is based almost entirely on computational modelling) may be unconservative as discussed previously. How, why, and where local buckling might occur and how this might affects global failure of a CFS column remain unclear[38], and the ability to accurately predict column failure modes is thus marginal.

5. Applied Fire Protection

Very little research is available on the use of intumescent coatings, which are by far the preferred method of fire protection, for these types of members. Well-validated models are not yet available to predict the evolution of material/thermal properties in the intumescent process; such models are needed for rational, performance-based design of these systems. Current interim guidance is therefore necessarily prescriptive[6]. Questions remain as to the consistency and uniformity of protection provided by intumescent systems that were developed for hollow or profiled sections on CFS columns and the consequences for structural performance in real fires where non-standard, localised, or non-uniform heating (and cooling) may occur.

5. Connections and Load Introduction

It is critically important in buildings incorporating CFS columns to ensure that loads from beams and floor plates can be transferred into the concrete core, both during ambient design and in design for fire. Various methods to accomplish load transfer in these members are available, including internal shear connectors or through plates. Very few studies are available on the heat transfer or structural performance of beam and floor plate connections to CFS columns during fire[39] and additional research is needed to identify robust, convenient, and economical beam-to-CFS column connections.

CONCLUSIONS & RECOMMENDATIONS

This paper has presented a review of available research, both experimental and computational, on the fire performance of CFS columns. It has highlighted a number of knowledge gaps that should be addressed to support fire safe and economical application of CFS columns in multi-storey construction. The most pressing areas identified for future research involve high strength FIB infilled CFS columns, both unprotected and protected (with intumescent paint), with flexural effects due to load eccentricity, secondary moments (slenderness), or imposed bending moments. Future research should also consider non-standard design fire scenarios relevant to performance-based designs.

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